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EFFECT OF INFILLS ON THE GLOBAL BEHAVIOUR OF R/C FRAMES: ENERGY CONSIDERATIONS FROM PSEUDODYNAMIC TESTS

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SUMMARY

A series of pseudo-dynamic tests were conducted on a full-scale four-storey reinforced concrete building designed according to Eurocodes 2 and 8. The building was 10 m long, 10 m wide, and 12·5 m high. It was designed as a ductility class 'High' structure, for typical live loads and for a peak ground acceleration of 0·3 g and medium soil conditions.

A first test was conducted on the bare frame. The project was carried out within the framework of the European Association of Structural Mechanics Laboratories (EASML), and was designed to assess the adequacy of the damage indicators to be used in the calibration of Eurocode 8. The pseudodynamic test was conducted by using an artificially generated earth-quake derived from a real earthquake (Friuli, 1976), with nominal acceleration 50 per cent larger than the value adopted in design. The structure performed as expected. The pattern of the measured rotations was that of a weak-beam, strong-column mechanism. The fundamental frequency of the structure after the test was found to be half of the initial value, but the damage was limited and uniformly distributed.

A second experimental programme was conducted as part of the work of the Network Prenormative Research in support of Eurocode 8, to study the influence of masonry infill panels on the global behaviour of the frame. Two pseudodynamic tests were conducted, with different infill patterns. A test was performed by infilling the two external frames with hollow brick masonry in all four storeys (uniform infill distribution). The test was then repeated on the structure without infills at the first storey, to create a soft-storey effect. The input signal was the same as in the tests on the bare frame. The purpose of the tests was to study the effects of the different layouts of infills, as well as to calibrate the computer models for the infills to be used in parametric analyses.

In this paper the test results are presented and the performances of the structure with different infill configurations are compared. The global behaviour of the structure is compared with the predictions which could have been made with simplified approaches. In particular, single degree of freedom energy concepts are used to verify if the differences in the global behaviour could have been predicted. The differences in the single degree of freedom energy demands with respect to the bare frame may be used as a means of accounting for the presence of irregular distributions of non-structural infills in the simplified design of the frame.

KEY WORDS: reinforced concrete; infilled frames; regularity; experimental methods; energy

INTRODUCTION

Modern seismic codes neglect, or take very limited account of, the effects of non-structural masonry panels. In fact, the masonry panels strongly affect the behaviour of the main structure. In general, the presence of non-structural masonry panels has a beneficial effect, because they significantly increase the global strength of the structure. On the other hand, they also increase the initial stiffness, so that the inertial forces may be increased to a large extent. The beneficial effect due to the increase of strength may or may not counterbalance the potentially negative effect due to the global stiffening of the structure.

Computer models are available to conduct parametric studies on the effects of the infill panels. Generally, phenomenological global models are used.[1] These models are of the equivalent diagonal strut type. They are

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simple and robust, but the calibration of the global properties is rather difficult. The experimental work conducted to-date[2] (generally on simple one-storey one-bay infilled subassemblages) does not provide data for the calibration of the global models, since the basic properties of the material — such as the strength of bricks and mortar, and of small wall specimens — are typically not reported.[1] To fill this gap, a more refined model is being developed at Ispra.[3] This includes 2-D smeared-crack elements for masonry and concrete, and either a full-bond or unilateral frictionless condition at the infill–frame interface. By means of a monotonic analysis, it is possible to calibrate the parameters required for the global models, starting from the basic properties of the materials.

Monotonic analyses of the test structure have shown that the results are extremely sensitive to the modelling assumptions. This confirms the need for the test to be conducted on the infilled frame, as well as the need for continuing the refinement of the computer models, to include effects such as friction at the interface.

An important problem concerning the effects of infills is their distribution. Irregular arrangement in plan and elevation may cause a significant concentration of damage in the frames, due to torsional effects or to the formation of soft-storey mechanisms. After the first test on the fully infilled structure, a second test was performed without infills in the first storey, to create a soft-storey effect with a local concentration of ductility demands. A companion testing activity was devoted to studying the importance of the out-of-plane response in the global behaviour.[4] It is believed that this experimental activity will allow the available computer models to be validated and calibrated, so that extensive parametric analyses can be carried out. The results of this study will assist in enabling the effects of the infills to be taken into account more realistically, something which could lead to an improvement in the codes.

TESTS PERFORMED ON THE BARE FRAME

Design of the specimen

The general layout of the full-scale reinforced concrete test structure (Figure 1) was decided during the EASML co-operative research.[5] It was a four-storey, high-ductility framed structure. Dimensions in plan were 10 m × 10 m, measured from the column axis. Inter-storey heights were 3·0 m, except for the ground storey (3·5 m). The structure was symmetric in one direction (that of testing), with two equal spans of 5·0 m, while in the other direction it was slightly irregular due to the different span lengths (6·0 and 4·0 m).

All columns had a square cross-section with a 400 mm side, except for the interior column which was 450 mm × 450 mm. All beams had a rectangular cross-section, with a total height of 450 mm and a width of 300 mm. A solid slab with a thickness of 150 mm was adopted for all storeys.

The materials used for the specimens were normal-weight concrete C25/30 as specified by Eurocode 2,[6] and B500 Tempcore (heat-treated steel with characteristic yielding strength 500 MPa), rebars and welded meshes. The adoption of this kind of steel was decided on because the Tempcore process is gaining the market in Europe. The B500 Tempcore steel does not fulfil the requirement for high-ductility structures given either by the 1988 edition of EC8[7] — used for the design of the test structure — or the version recently issued as prenormative standard (ENV).[8] The minimum characteristic value of the deformation at maximum stress is fixed as 9 per cent in the ENV — this was prescribed in terms of strain at failure in the 1988 version — whereas the actual value is 8·5 per cent. The lower bound of the mean value of the tensile strength to yield strength ratio is fixed as 1·20 in the ENV — this was 1·30 in the 1988 version — whereas the actual value is 1·16. The results of the test were expected to provide information about the adequacy of this material for use in earthquake resistant construction.[9]

The preliminary design was carried out in accordance with the prescriptions of Eurocodes 2 and 8, assuming typical loads (additional dead load 2·0 kN/m^2, to represent floor finishing and partitions, and live load 2·0 kN/m^2), and high seismicity. The seismicity was defined by the elastic spectrum defined in EC8, with peak ground acceleration of 0·3 g and medium soil conditions (soil type 'B' in EC8).

EC8 allows for the choice of three different ductility classes (Low, Medium and High), corresponding to different values of the force reduction factor (q), and of different design methods according to the structural regularity. The structure satisfied the regularity requirements both for vertical and horizontal configuration,

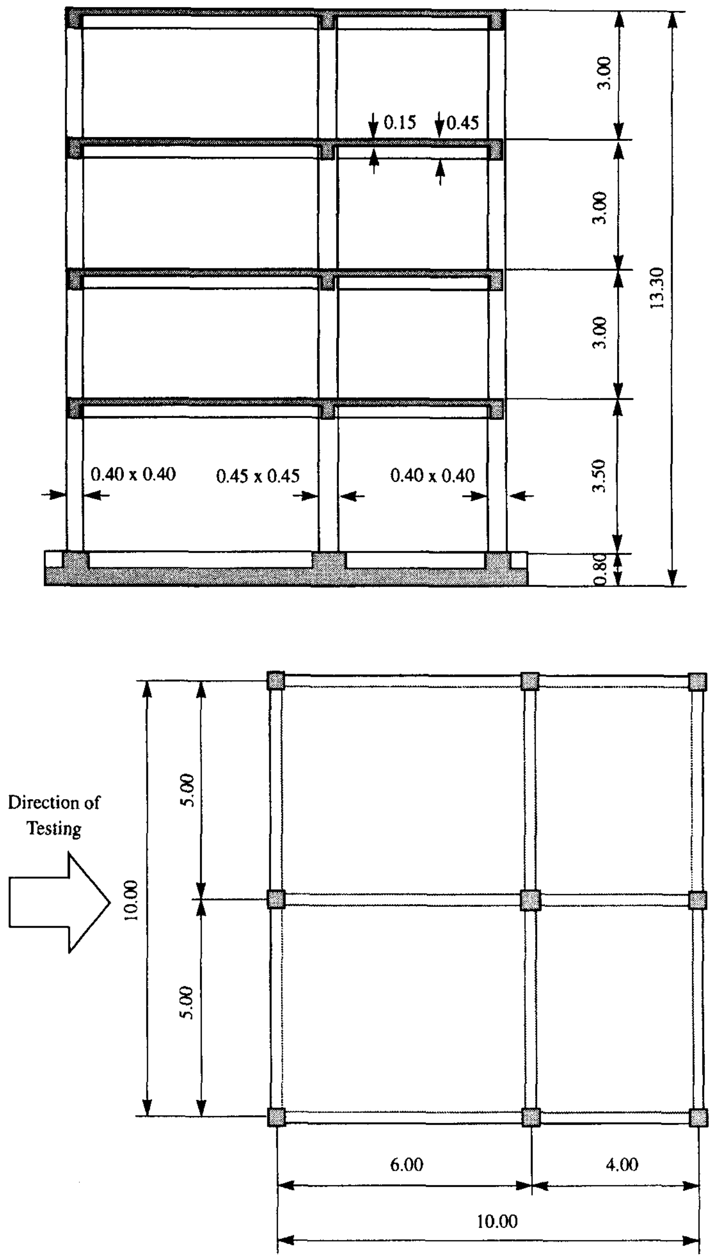


Figure 1. Layout of the specimen (dimensions in metres)

so it was designed as a frame of high regularity in ductility class High, with the behaviour factor $q = 5$. The design was performed using two independent planar models for each orthogonal direction, and the torsional effects were taken into account by the simplified method prescribed by EC8.

The preliminary evaluation of the fundamental period yielded a value of $T = 0{\cdot}53$ sec, which corresponds for the soil type B to the flat region of the design spectrum. This yielded a total horizontal design force of 529 kN. The computed drifts, conventionally calculated according to the 1988 version of EC8, proved smaller than the limit fixed for non-structural elements 'fixed in such a way as not to interfere with structural deformations', but greater than the limit for non-structural elements 'of brittle materials attached to the structure'.

All members were detailed in accordance to the rules of the 1988 version of EC8. As far as practical detailing is concerned, it was decided to place the splicing of the bars at the columns of the ground floor in correspondence to their mid-heights.This solution is not adopted in practice in most cases, but it is considered by far the most rational and it allowed a simpler interpretation of the results. Details of the reinforcement layout of the frame are reported elsewhere.[10] The structure was loaded by means of two double-acting, 500 kN hydraulic actuators per storey, plus one extra actuator placed orthogonally at the third floor to prevent unwanted sway motion. Details about fixtures and control instrumentation are given in Reference 10.

The high-level pseudodynamic test

An artificial accelerogram, derived from the 1976 Friuli earthquake, was adopted. The signal was generated to fit the EC8 elastic response spectrum with a peak acceleration of 0·3 g. The reference signal is shown in Figure 2, and the corresponding 5 per cent damping elastic response spectrum is given in Figure 3, together with the spectrum specified by EC8 for 0·3 g and medium soil conditions. The test was performed

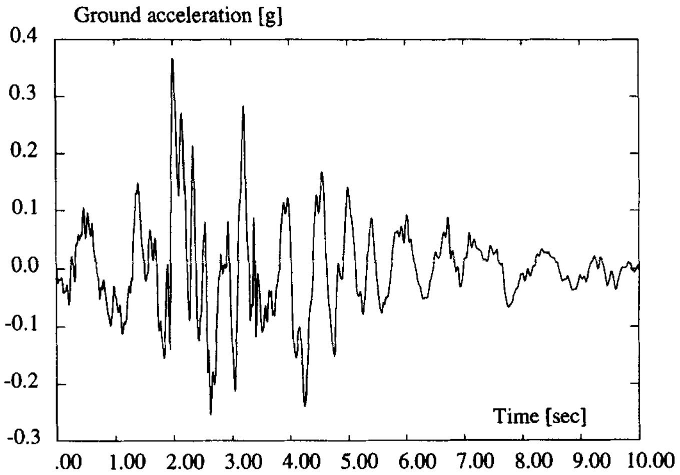


Figure 2. Reference Friuli-like generated accelerogram

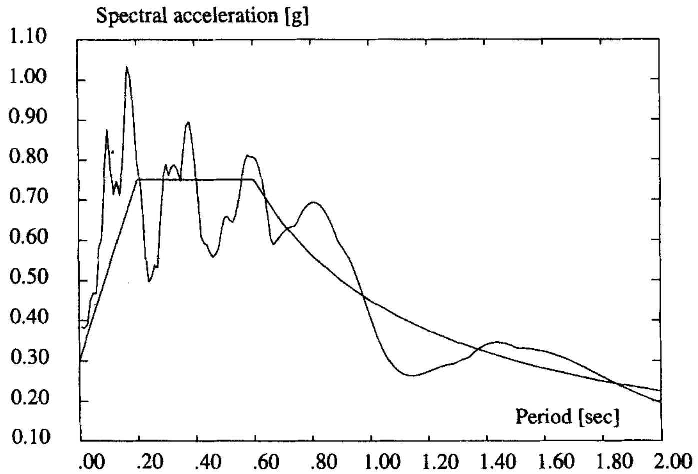


Figure 3. Reference accelerogram elastic spectrum and EC8 elastic spectrum

using the reference signal multiplied by an intensity factor of 1·5. The corresponding EC8-equivalent peak ground acceleration was then $1{\cdot}5 \times 0{\cdot}3 = 0{\cdot}45\, g$, a value which was thought to be representative of the maximum seismic actions for which the frame had been designed, accounting for possible sources of overstrength such as the overstrength from torsion which was considered in the design but not in the test. This intensity level is particularly meaningful in defining the damage resulting from the design-level seismic actions. A quantitative assessment of the damage suffered by the structure will be possible only when experimental data about the ultimate strength and ductility of the frame becomes available. To this aim, a final test to failure will be performed after the other tests scheduled for this specimen. After that stage it will be possible to draw definite conclusions about the performance of the structure. Post-test calculations must also be performed to understand fully the behaviour of the structure. Data already available do, however, allow some important conclusions to be stated.

The structure performed very well. During the test, cracks opened (and closed) in the critical regions of the beams of the first three storeys and of most of the columns. Only the cracks at the beam-to-column interfaces remained permanently open. Neither spalling of the cover nor local instabilities of reinforcement was observed. Besides the cracks at the beam-to-column interface, which were apparent in the first three storeys and represented evidence of local yielding in the rebars, the specimen remained apparently undamaged.

A direct stiffness measurement was performed before and after the pseudodynamic test. With the stiffness matrices, the vibration eigenfrequencies were computed. The resulting fundamental frequency (0·82 Hz) was less than half the size of that of the virgin structure (1·78 Hz). The resulting mode shapes, however, were close to those of the virgin structure. The fact that the mode shapes did not significantly change, even though the changes in the stiffness matrix of the specimen were significant, provide evidence that the structure was uniformly damaged. The analysis of the maximum rotations measured at the potentially critical locations (Figure 4) confirms the more or less uniform damage pattern (with the exception of the top storey), even though no plastic hinges appeared to form in the beams of the external frame at the intersection with the central column of the second storey.

Measurement of the rotations of the beams at different distances from the joint highlighted the important role played by the slippage of the rebars in the internal joints. This effect was probably made more severe by the adoption of the Tempcore steel. It resulted in significantly pinched force–displacement loops, and was found to be responsible for most of the differences with respect to the preliminary calculation.[11,12]

Details on the tests performed on the bare frame can be found in Reference 10.

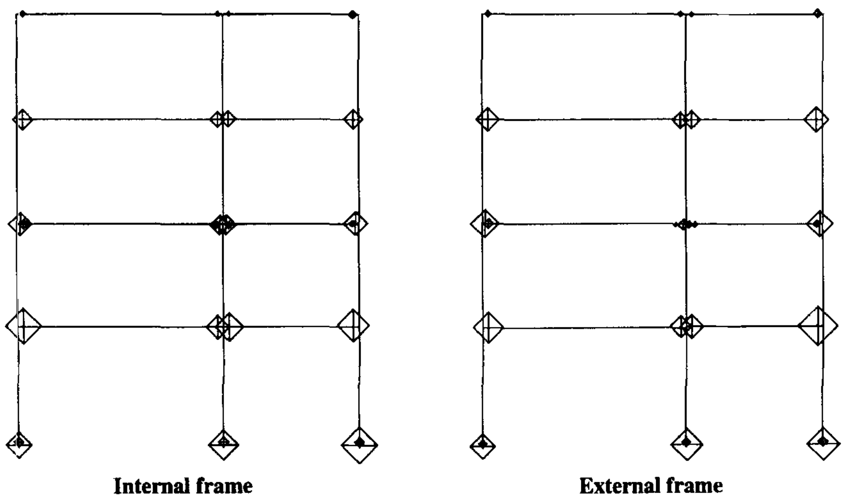


Figure 4. Test on the bare frame: maximum measured rotations (max 14·9 mRad)

TESTS ON THE INFILLED FRAME

Due to the relatively low damage suffered by the structure as a result of the bare-frame tests, no repair actions were taken. The materials to be used for the construction of the infill panels were selected as representative of typical light non-structural masonry. To facilitate the construction, blocks commonly available in Italy were adopted. The blocks had dimensions of $245 \times 112 \times 190\,(h)$ mm, with vertical holes taking 42 per cent of the gross section. The average compressive strength of the blocks was found to be, respectively, 13·3 MPa in the direction parallel to the holes (vertical), and 3·3 MPa in the direction orthogonal to the holes. The mortar was also selected as typical, to reach a compressive strength of 5 MPa (cement, lime and sand in the proportion 1 : 1 : 5).

To provide the data necessary to calibrate the global computer models, tests on small specimens were performed at the University of Pavia to measure the resulting properties in the strong and weak directions, as well as in the diagonal direction.[13] For the strong (vertical) direction, a mean strength of 7·3 MPa and a mean modulus of elasticity of 8210 MPa were obtained (values referring to the gross area). The compressive tests in the direction orthogonal to the holes yielded a mean strength of 2·4 MPa and a modulus of elasticity of 2515 MPa. A mean value of the tensile strength of 0·28 MPa and a shear modulus of 1240 MPa were derived from the results of the diagonal compression tests.

Uniformly infilled frame

A pseudodynamic test was conducted on the four-storey R/C frame with infills at all storeys (Figure 5) to appraise its seismic response as compared to the bare-frame behaviour. No special provisions were made in the construction of the panels, so that complete bond between the panels and the frame could be assumed at the beginning of the test. No plaster was placed on the surfaces of the panels.

The loading apparatus was the same as in the test on the bare frame. Before the test, the direct measurement of the stiffness matrix was carried out by displacing each of the storeys in turn by a small quantity, while holding the others still. From the stiffness matrix, the corresponding elastic vibration frequencies were computed. The fundamental frequency was found to be 3·34 Hz (for the bare frame this was 1·78 Hz before the test, and 0·82 Hz after the high-level test). Due to the fragile behaviour of the mortar

Figure 5. Uniformly infilled frame

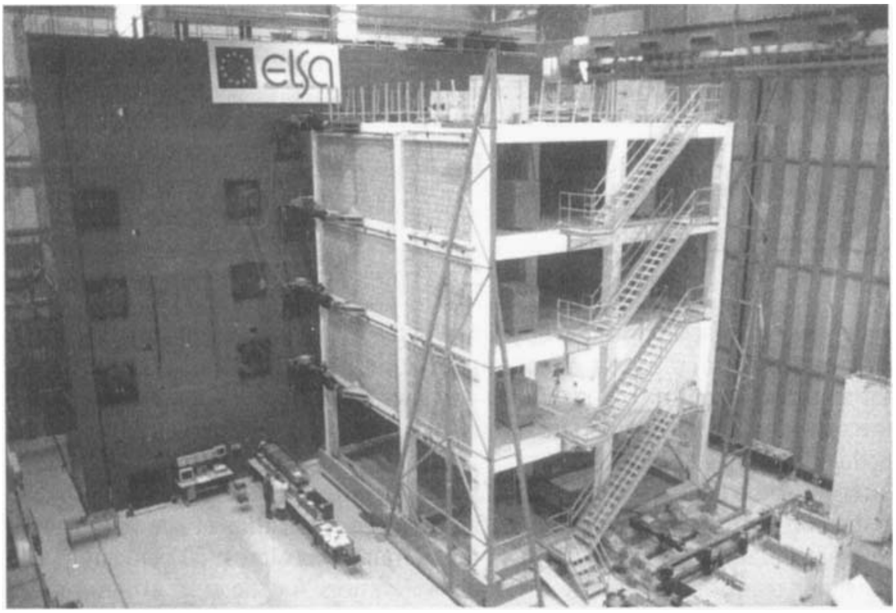


Figure 6. Soft-storey infilled frame

interface, no low-level tests were performed. A high-level test, with the same base motion as for the test on the bare frame (0·45 g nominal base acceleration) was carried out.

The test led to the complete destruction of the masonry panels at the first and second storeys. The panels at the third storey suffered extensive damage, and the ones at the upper storey remained almost intact. Redundant measurements were taken at each panel up to failure, to assist in the calibration of the global computer models to be used in the interpretation calculations. The global measured quantities will be presented in the following sections and compared with those from the other tests.

Soft-storey frame

The masonry panels were demolished and replaced with new ones, leaving the first storey bare (Figure 6). This was expected to lead to a soft-storey mechanism, and corresponds to the case of 'drastic reduction if infills in one or more storeys' in Eurocode 8. For such cases, the code requires a local increase of the forces to be used in design, as well as an increase in the portion of the columns of the ground floor to be detailed as for critical regions. up to the entire length of the column. However, none of these requirements was considered in design.

The fundamental frequency from the stiffness measurements proved to be 1·6 Hz. The high-level test was repeated as for the uniformly infilled frame.

As expected, the test resulted in a concentration of drift at the ground floor, and some damage was suffered by the panels of the second storey only. As for the previous test, local measurements were taken at some of the masonry panels. In addition, the rotations at the ends of the columns and beams of the ground and first floors were monitored. The global results will be discussed in the next section.

DISCUSSION OF THE RESULTS

General considerations

The comparison of the results obtained in the tests with different infill configurations (Table I) is particularly meaningful, because most of the design codes neglect the changes due to the presence of the infills.

Table I. Summary of measured quantities from the three tests

Structure	Frequency (Hz)	Spectral acceleration (g)	Max top displacement (mm)	Max base shear (V_0/W)	Max drift (%)	Max beam rotation (mrad)	Max column rotation (mrad)
Bare frame	1·78	1·11	210	0·40	2·4	13·9	14·9
Uniformly infilled	3·30	1·16	80	0·62	1·1	—	—
Soft storey	1·66	1·19	180	0·47	3·5	15·0	32·6

The measured storey displacements are shown in Figure 7. The maximum top displacement obtained for the bare frame (about 210 mm) proved to be comparable with the one obtained for the soft-storey structure, even though the spatial distribution of the storey drifts was obviously quite different. In spite of the almost complete failure of the infill panels, the maximum top displacement experienced by the uniformly infilled frame was less than two-fifths the size.

The time histories of the base shear obtained for the three tests are depicted in Figure 8. The maximum base shear for the soft-storey structure is only slightly larger than that of the bare frame (1·45 MN). The maximum base shear obtained for the uniformly infilled frame was almost 50 per cent larger than that of the bare frame. It is worth mentioning that the spectral accelerations for the three structures were almost equal.

The storey-level hysteretic loops (Figure 9) provide some insight into the behaviour of the structure. The storey shear vs. inter-storey drift diagrams for the bare frame exhibit stable dissipative loops, with amplitude progressively decreasing from the first to the top level. Even though the loops take a pinched shape after the first large-amplitude cycle — the main reason for this effect having been identified as the loss of bonding in the internal joints — no strength deterioration occurs. For the case of the uniformly infilled frame, the amplitude of the cycles also decreases with the storey level. However, severe strength and stiffness deterioration — typical of infill behaviour — can be noticed. In the case of the soft-storey frame, the energy dissipation is largely limited to the ground floor, and the onset of pinching in the shape of the loops after the first amplitude cycle is as evident as in the loops of the bare frame.

The inter-storey drift profiles are given in Figure 10. The values for the uniformly-infilled frame are less than half of those of the bare frame. In the bare frame the largest value was found at the second storey instead of the first storey, while the opposite applies to the tests on the infilled frame. For the soft-storey structure, most of the interstorey drift took place at the bottom storey, where a value larger than 3·5 per cent was achieved.

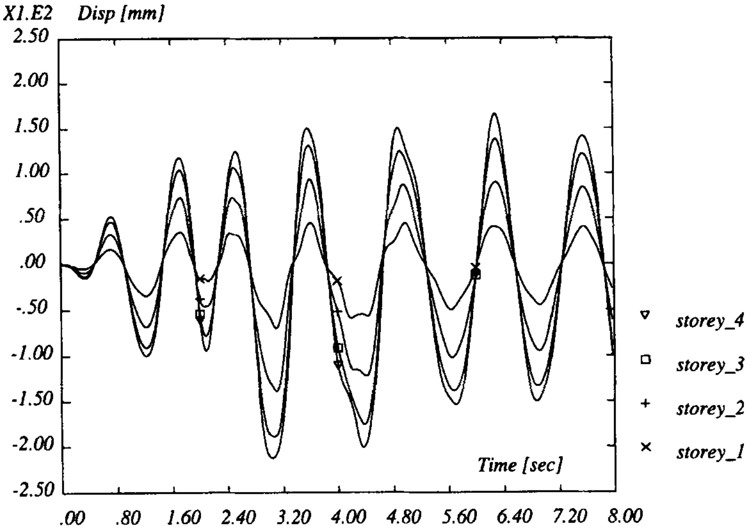
Damage suffered in the various infill configurations

The testing activity on the bare frame was started to assess the applicability of various damage indicators. In particular, the aim was to compare the effectiveness of the proposed mechanistic damage indices with widely used damage indicators (such as maximum ductility, inter-storey drift, global stiffness degradation).

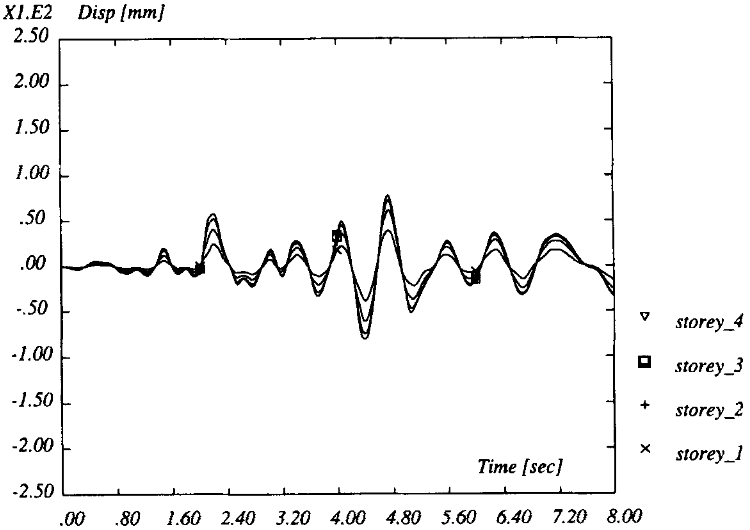
The assessment of the mechanistic damage indices will become possible only after the final cyclic test to failure predicted for the bare frame, after the necessary repair. This will enable the ultimate displacement capacities, as well as the sensitivity to cyclic damage, to be estimated, so that the equivalent monotonic ultimate displacement can be determined.

Local rotation or curvature ductilities cannot be derived from the test data, since no internal force measurements were made. Global storey ductilities can be derived from the global storey hysteretic loops, but the comparison is of little significance. In the case of the uniformly infilled frame, the non-linearity is substantially due to the damage in the masonry panels. In the case of the soft-storey infilled frame, the global ductility demand at the bottom storey is much larger than in the bare frame. However, the two quantities are not directly comparable as damage indicators, since that of the bare frame is associated

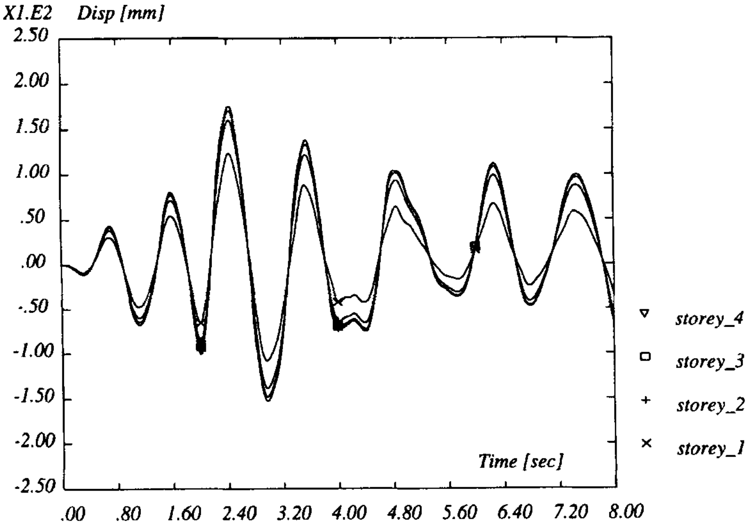
Figure 7. Measured storey displacements



Bare frame



Uniformly infilled frame



Soft-storey infilled frame

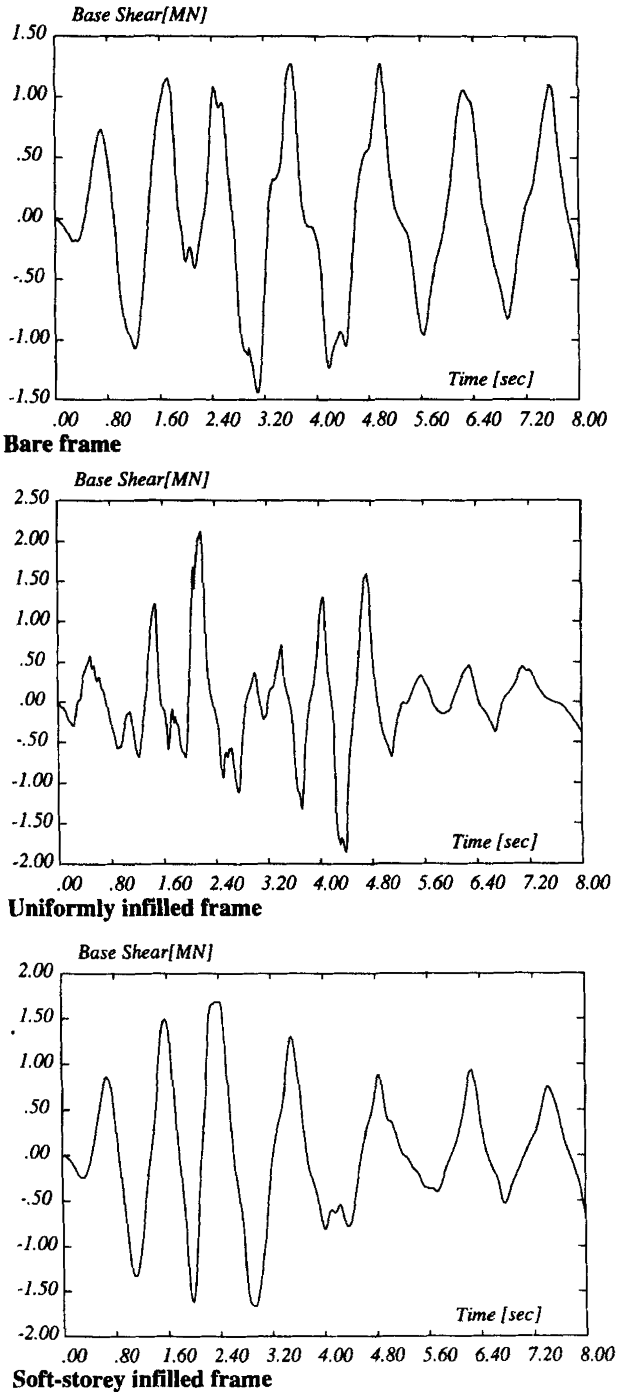


Figure 8. Time histories of base shear

with a strong-column, weak-beam mechanism, while that of the soft-storey infilled frame is associated with a bottom-storey local sidesway mechanism, as can be seen from the maximum measured rotations (Figure 11). Due to the different dissipative mechanisms activated in the tests, the global ductilities are not related to the member-level ductilities in the same fashion.

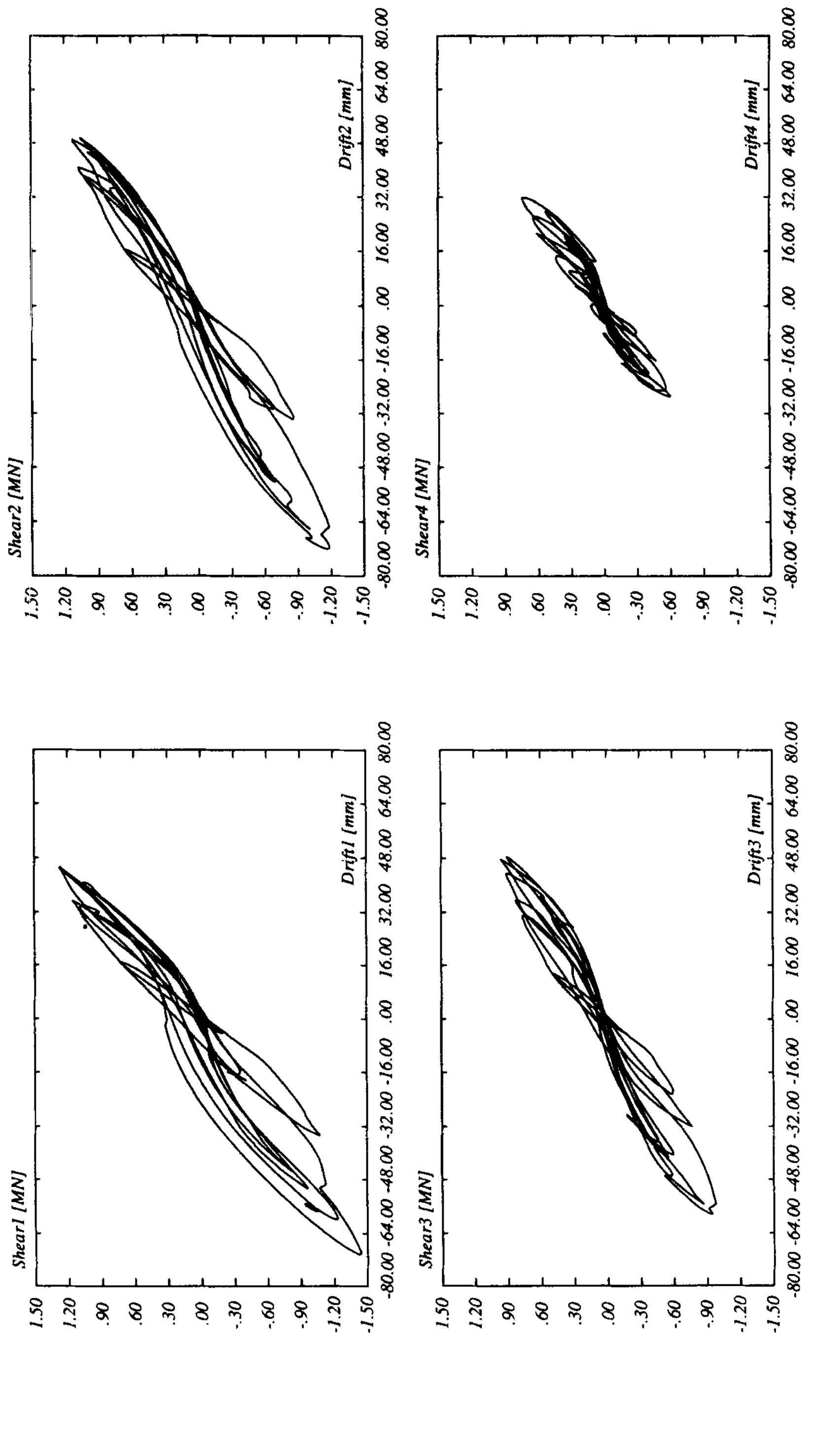


Figure 9. Storey-level hysteretic loops: (A) Bare frame

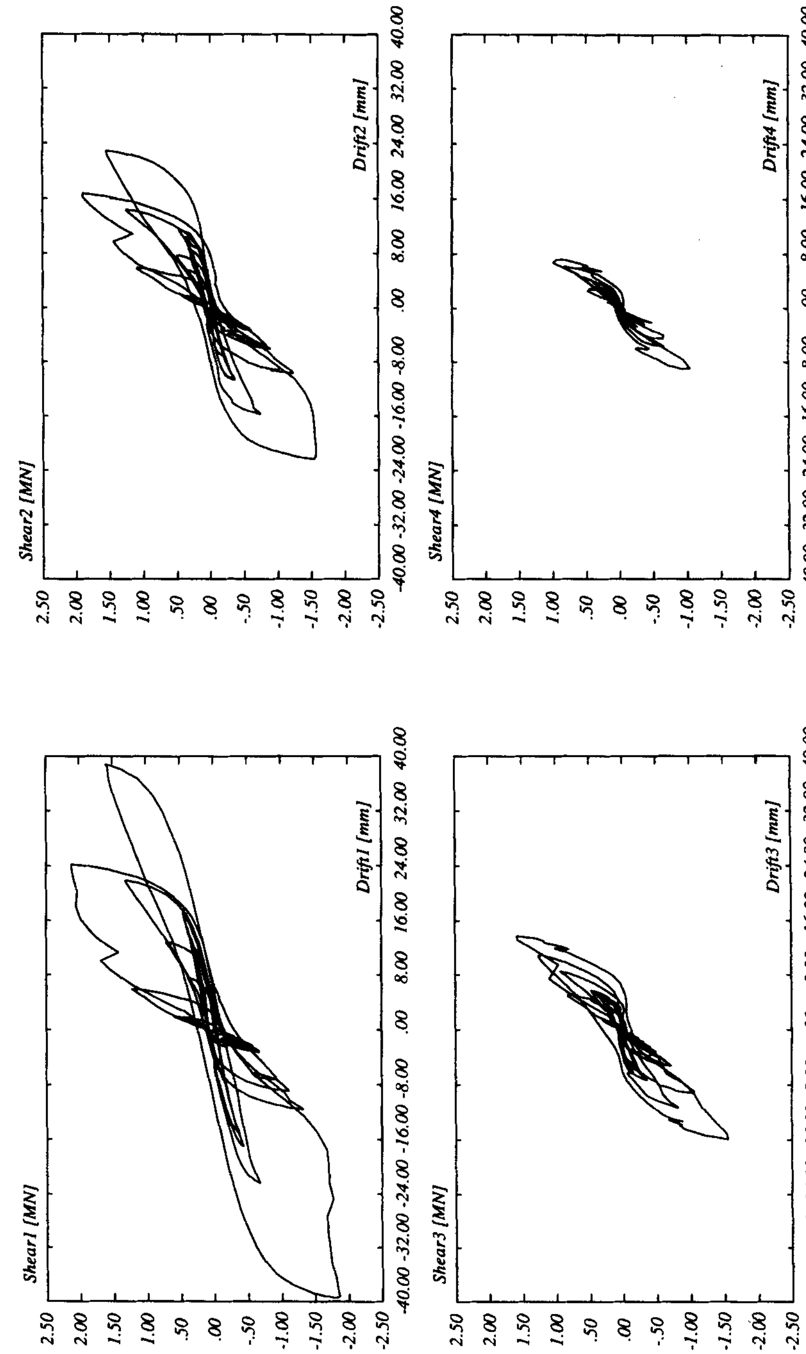


Figure 9. (B) uniformly infilled frame

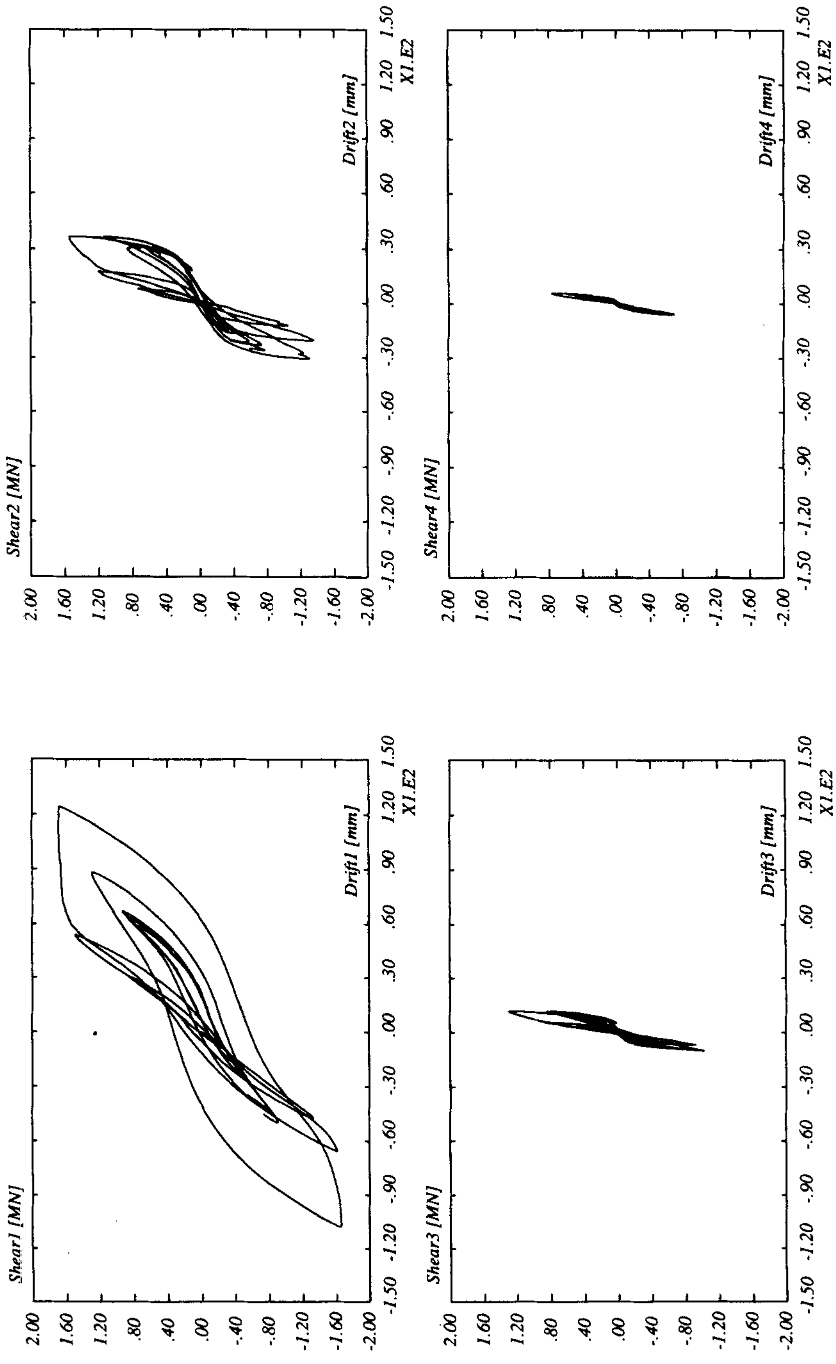


Figure 9. (C) soft-storey infilled frame

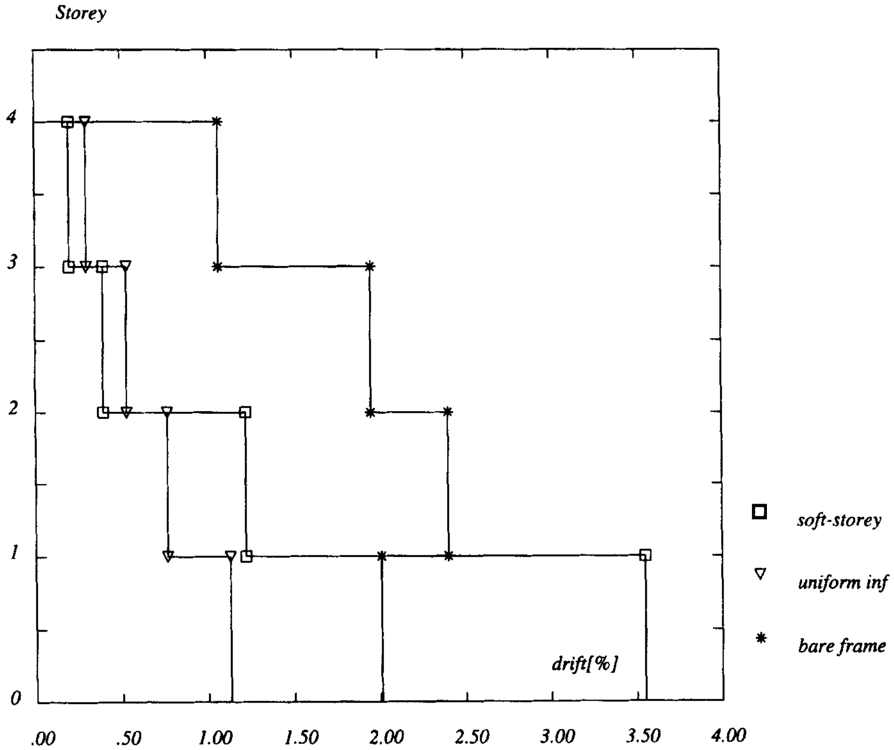


Figure 10. Maximum inter-storey drift profiles

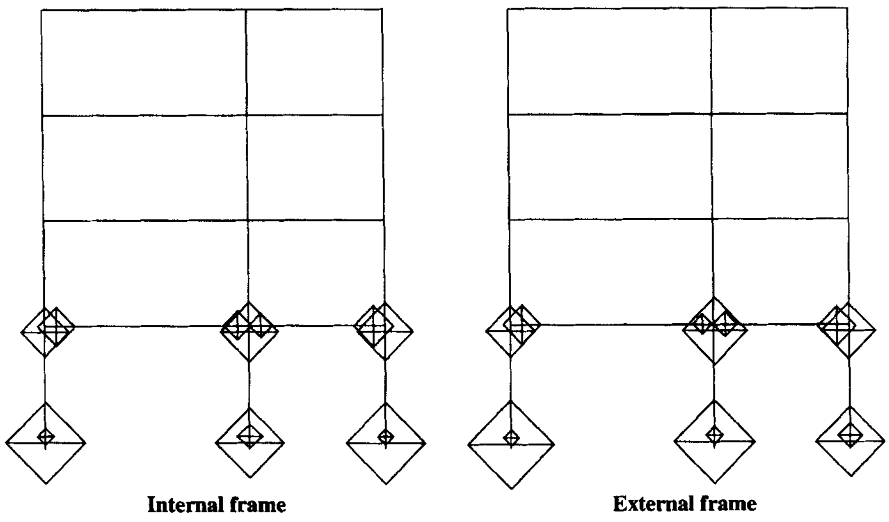


Figure 11. Test on the soft-storey infilled frame: maximum measured rotations (max 32·6 mRad)

It is difficult in this case to relate the maximum inter-storey drifts to the damage suffered by the structure. The maximum inter-storey drifts measured during the tests (Figure 10) are 1·13 per cent for the uniformly infilled frame, 2·4 per cent for the bare frame and 3·55 per cent for the soft-storey infilled frame. The largest values, however, were found at the bottom storey for the infilled frame, and at the second storey for the bare frame. It is interesting to note that the ratio of the maximum rotations measured at the member ends of the

bottom storey for the tests on the infilled frame and on the soft-storey infilled frame is not comparable with the ratio of the corresponding maximum drifts. This is further evidence for the different dissipative mechanisms activated during the tests.

Stiffness degradation can be regarded as a damage indicator. The global stiffness matrices were evaluated from the on-line measurements by using the procedure developed by Igarashi et al.[14] This procedure is based on a second-order updating of the initial stiffness matrix, similar to the BFGS[15] stiffness updating method used in the solution of non-linear equations with quasi-Newton strategies.[15] The initial values of the stiffness matrix were measured via direct stiffness measurement before the test, and the update procedure was applied continuously by using the displacements and restoring forces measured during the test. The resulting time histories of the vibration frequencies are given in Figure 12. The progressive deterioration of stiffness is noticeable. Moreover, the final values of the frequencies are in agreement with the frequencies computed from the stiffness matrix directly measured after the test. However, the effects of the failure of the infill panels are not apparent.

Energy can also be used as an indicator of the expected damage.[16] This will be addressed in the following section.

Energy considerations

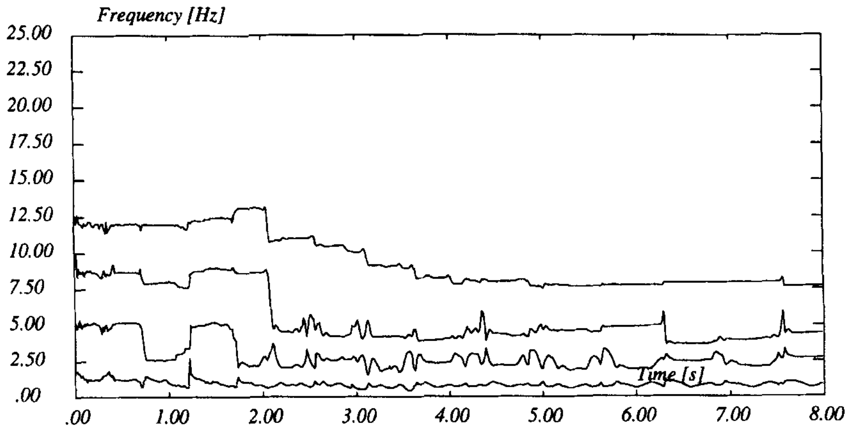
Absorbed energy from test results. The time histories of the total absorbed energy at each storey are depicted in Figure 13 for the three tests. The total absorbed energy represents the sum of elastic strain energy and hysteretic energy. However, it is apparent from the figure that the elastic energy represents a small portion of the total absorbed energy.

From Figure 13 one can understand how the energy dissipation was distributed among the different storeys. In the bare-frame structure, most of the energy dissipation took place in the first two storeys, with more or less the same amount of dissipation. The energy dissipated at the third storey was still noticeable, while the energy dissipated at the top storey was almost negligible. The effect of the infills in the uniformly infilled structure was to reduce further the higher storeys' contribution to energy dissipation, though they still contributed significantly to the total energy dissipation. For the soft-storey infilled structure, almost all the energy dissipation took place at the bottom storey.

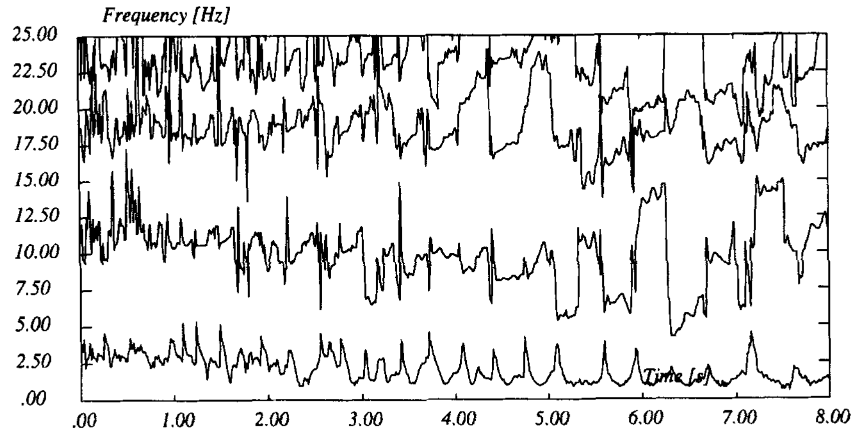
The large-amplitude drops in the graphs correspond to the points where the individual masonry panels failed. It is interesting to recall how this piece of information could not be found in the time histories of the vibration frequencies.

In Figure 14 the total absorbed energies for the complete structure are given. Since no viscous damping was specified during the test, the total absorbed energy is equal to the difference between the input energy and the kinetic energy. The analysis of the input energy provides the opportunity to compare the test results with the energy demands which could have been predicted by using simple SDOF considerations. The need to check the validity of applying the results obtained by SDOF analysis of MDOFs by means of experiments on whole multi-storey buildings has already been expressed.[16] If this comparison were satisfactory, energy considerations could represent a simple way to account for the presence of the infills in design, for instance by modifying the design forces according to the different input energy demands with respect to that of the bare frame.

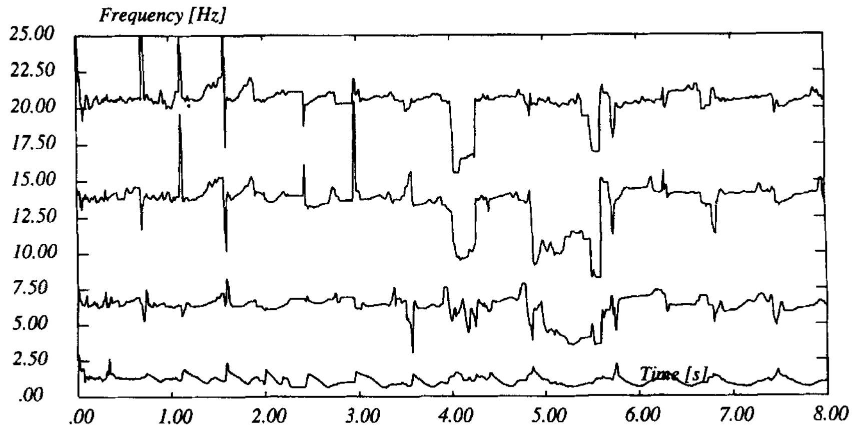
Housner's assumption. A simple way of using SDOF calculations to predict the energy demand for an MDOF structure is to use Housner's assumption.[17] According to this assumption, one can estimate the energy demand as the maximum kinetic energy of the corresponding linear SDOF system. Since we have already noticed that the spectral pseudo-acceleration is roughly the same for the three structures — approximately 1·2 g for the high-level earthquake — one can easily compute the kinetic energy of the linear SDOF system corresponding to each of the three structures. The effective masses were obtained from the eigenvectors of the measured stiffness matrices. The energy demands obtained for the bare frame, the uniformly infilled structure and the soft-storey infilled structure are, respectively, 169, 41 and 224 kJ. These values compare unfavourably with the total absorbed energies at the end of the tests (Figure 14). This is further evidence that the energy demands computed by using Housner's assumption tend to be underestimated.[18]



Bare frame



Uniformly infilled frame



Soft-storey infilled frame

Figure 12. Time histories of frequencies from tangent stiffness updates

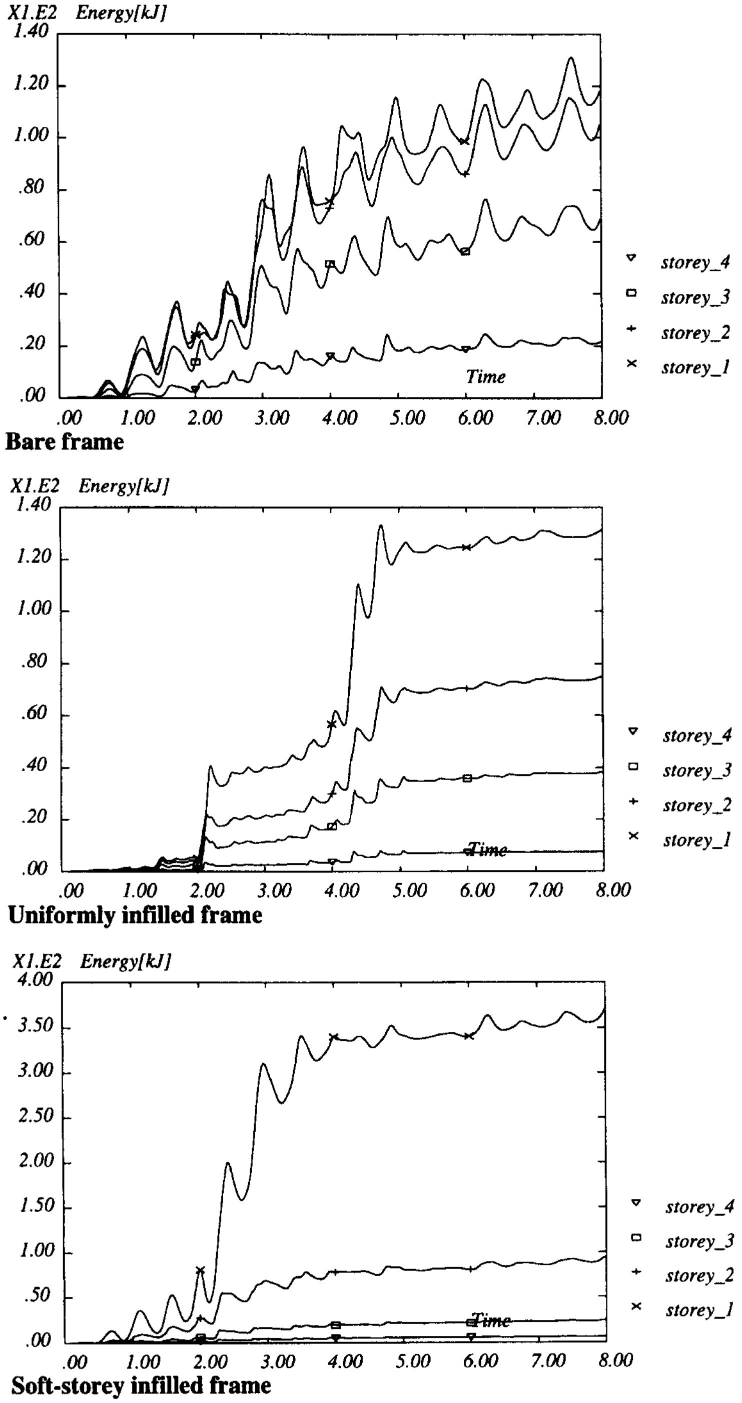


Figure 13. Time histories of storey-level absorbed energy

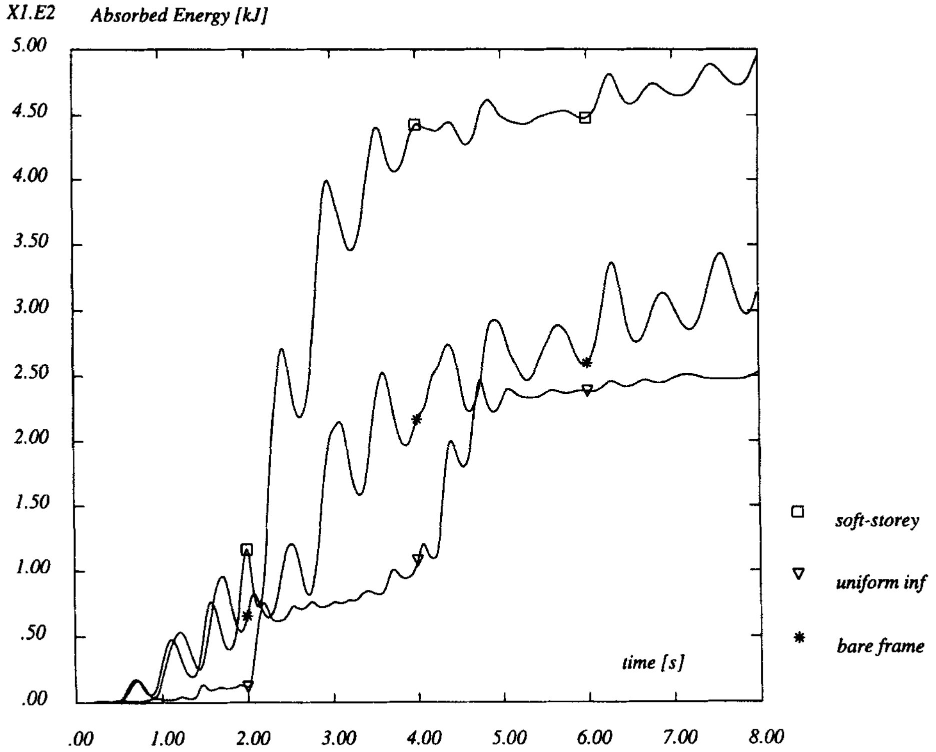


Figure 14. Time histories of total absorbed energy

Absolute vs. relative energy. Uang and Bertero have shown that absolute input energy based on an elastic perfectly plastic SDOF system can provide a good estimate of the energy demands for a multi-storey building.[18] Absolute energy is the energy evaluated with reference to the absolute system, i.e. including energy terms related to ground motion, while relative energy is evaluated in the equivalent fixed-base system. To clarify the differences between the two definitions, it is useful to recall the equation of motion for the SDOF subjected to ground motion:

$$m\ddot{v}_t + c\dot{v} + f_S = 0 \tag{1}$$

where m is the mass of the system, c the viscous damping coefficient, f_S the structural restoring force, v_t the total displacement of the mass and v the relative displacement of the mass with respect to the ground. By introducing the notation $v_t = v + v_g$, where v_g is the input ground motion, equation (1) becomes

$$m\ddot{v} + c\dot{v} + f_S = -m\ddot{v}_g \tag{2}$$

By integrating equation (1) with respect to v, we obtain the 'absolute energy equation'

$$E_i = E_k + E_c + E_a \tag{3}$$

The first term on the right is the 'absolute kinetic energy'

$$E_k = \frac{m\dot{v}_t^2}{2} \tag{4}$$

the second term is the damping energy

$$E_c = \int c\dot{v}\,dv \tag{5}$$

the third is the absorbed energy

$$E_a = \int f_S \, dv \tag{6}$$

and the term on the left is the 'absolute input energy'

$$E_i = \int m\ddot{v}_t \, dv_g \tag{7}$$

Analogously, by integrating equation (2) with respect to v one obtains the 'relative energy equation'

$$E_i' = E_k' + E_c + E_a \tag{8}$$

The second and third terms on the right remain unchanged with respect to the absolute energy equation. The first term on the right represents the 'relative kinetic energy', i.e.

$$E_k' = \int m\ddot{v} \, dv = \frac{m\dot{v}^2}{2} \tag{9}$$

and the term on the left is the 'relative input energy':

$$E_i = -\int c\ddot{v}_g \, dv \tag{10}$$

It has been demonstrated that the relative input energy is practically independent of the ductility demand and the damping, and the energy spectra for non-linear systems are very similar to those of linear systems.[9] According to Uang and Bertero the absolute input energy is also practically independent of the ductility demand,[16] except for the case of signals with extremely long duration. These facts should make the use of energy concept a valuable tool for assessing the differences in the global behaviour of the same structure in different infill configurations.

The limitations of the SDOF procedures in dealing with structures exhibiting soft-storey mechanisms have already been reported.[20] However, the need to check the validity of applying the results obtained by SDOF analysis to MDOFs by means of experiments on whole multi-storey buildings has also been expressed.[16] The results of the tests performed on the frame with different infill configurations will be used for this purpose. If SDOF energy considerations were sufficient to highlight the differences in the global behaviour of infilled frames, the differences in the input energy demands with respect to the bare frame may represent a simple way to account for the presence of irregular infill distributions in design.

A constant-ductility elastic perfectly plastic response spectrum was constructed by iterating on the strength-reduction factor until the reference ductility was obtained, with a 1 per cent tolerance, and both absolute input energy and relative input energy were computed according to the definitions given in equations (7) and (10). This was done for a target ductility value of 5, which was the global ductility assumed in design for the bare frame. The choice of the ductility value was not expected to yield large differences in the input energy, due to the substantial independence of the input energy from the ductility claimed by different authors.[19,16] For the same reason, zero damping was assumed in constructing the response spectrum. The resulting input energies — absolute and relative — are given in Table II, together with the input energies computed for the elastic SDOF system, and they are compared with the measured input energies. The measured input energies were computed for the four-DOF system as the sum of absorbed and — absolute or relative — kinetic energy.

From Table II, it can be concluded that neither the relative nor the absolute SDOF input energy demands reflect the measured input energy. The assumption that the input energies are independent of the ductility level was not verified. The comparison is more satisfactory for the elastic SDOF energies. Absolute input energies compare relatively better than relative input energies.

Even though the comparison of the SDOF analysis and the test results was not satisfactory, and further studies are probably needed to clarify the importance of the ductility demand in the SDOF input energy, we

Table II. Computed values of absolute (E_i) and relative (E_i') input energy (kJ)

Structure	Frequency (Hz)	Effective mass (kg × 10^3)	SDOF $\mu = 5$		Elastic SDOF		Measured	
			E_i	E_i'	E_i	E_i'	E_i	E_i'
Bare frame	1·78	295	366	353	444	400	398	377
Uniformly infilled	3·30	275	164	117	269	184	303	258
Soft storey	1·66	337	401	379	552	509	536	511

can still conclude that the SDOF input energy was sufficient to highlight the differing global structural behaviour for the different infill configurations. From the SDOF input energies one could in fact be forewarned about the substantially different energy demands for the uniformly infilled frame and the soft-storey infilled frame with respect to the bare frame.

Since no simple, yet accurate, methods exist to account for the distribution of non-structural infills in moment resisting frames, a method based on the simple modification of the design forces according to the differences in the SDOF energy demand with respect to the bare frame should be explored.

CONCLUSIONS

The global results of the pseudodynamic tests conducted on a full-scale four-storey reinforced concrete frame designed according to Eurocodes 2 and 8 have been presented. The tests were conducted on the bare frame, as well as on the frame with two different configurations of non-structural masonry infills.

The effects of the infills on the global behaviour of the structure have been discussed, and the response of the structure in the three configurations has been compared with the predictions which could have been made by means of simplified SDOF techniques. The following conclusions can be stated:

(1) The presence of light non-structural masonry infills can change the response of the structure to a large extent. The presence of a regular pattern of infills to a large extent prevents energy dissipation from taking place in the frame (this was true in spite of the progressive complete failure of the panels at the first two storeys). Irregularities in the panels result in unacceptably larger damage to the frame. In general, the effect of the non-structural infill panels cannot be neglected in design.

(2) The differences in the behaviour of the structure with different distributions of infills can be captured by means of simplified SDOF techniques based on energy considerations. The differences in the SDOF energy demands with respect to the bare frame may be used as a means to account for the presence of irregular distributions of non-structural infills in the simplified design of the frame.

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